Energy, Mines and Resources Canada | Énergie, Mines et Ressources Canada

CANMET

Canada Centre for Mineral and Energy Technology | Centre canadien de la technologie des minéraux et de l'énergie

STANDARD TRIAXIAL COMPRESSION TESTS ON WN1 GRANITE CORE SPECIMENS

P. MILES

ELLIOT LAKE LABORATORY

MARCH 1979

MINERALS RESEARCH PROGRAM

MINING RESEARCH LABORATORIES
REPORT MRP/MRL 79-18 (TR)

STANDARD TRIAXIAL COMPRESSION TESTS ON WN1 GRANITE CORE SPECIMENS

by

P. Miles*

SUMMARY

Standard triaxial compression tests were performed on six WN1 granite specimens from the Pinawa test site, Pinawa, Manitoba, as part of CANMET's involvement in the underground nuclear waste repository program.

Tests on the 44.94 mm diameter specimens were carried out at confining pressures of 3.45 MPa, 17.24 MPa and 34.48 MPa. Corresponding to these confining pressures the average triaxial strengths were 262.14 MPa, 451.77 MPa and 555.08 MPa. Thus, fivefold and tenfold increases in confining pressure resulted in strength increases of 1.7 fold and 2.1 fold respectively. The mean tangent modulus of elasticity, determined at 50% ultimate strength, was $68.94 \pm 3.70 \times 10^3$ MPa and the mean Poisson's ratio was 0.41 ± 0.10. Mohr's circles and the corresponding strength envelope were constructed and associated cohesion and angle of internal friction values as well as failure plane shear and normal stresses were determined.

---

* Engineer, Elliot Lake Laboratory, Mining Research Laboratories, Canada Centre for Mineral and Energy Technology, Department of Energy, Mines and Resources, Elliot Lake, Ontario.

CONTENTS

Summary ---------- i
Introduction ---------- 1
Laboratory Testing ---------- 1
Test Specimens ---------- 1
Procedures ---------- 2
Results ---------- 4
Acknowledgements ---------- 9
References ---------- 9

TABLE

1. Results of Triaxial Compression Tests on WN1 Granite Specimens 3

FIGURES

1. Stress-strain curves for WN1 granite specimens at various confining pressures ---------- 5
2. Increase in triaxial compressive strength with confining pressure ---------- 6
3. Mohr's circles and the strength envelope ---------- 8

INTRODUCTION

Part of CANMET's contribution to the underground nuclear waste repository program involves the laboratory testing of hard rock samples from potential repository sites in Canada. The testing objective is to provide design information aiding in a ranking of potential sites on the basis of the rocks' mechanical and thermal properties.

Six WN1 granite specimens were received from the Pinawa test site, Pinawa, Manitoba and standard triaxial compression tests were carried out to determine their mechanical properties. Testing was done at the Mining Research Laboratories, Elliot Lake Laboratory, Elliot Lake, Ontario.

The following report provides details on sample preparation, test procedures and obtained results.

LABORATORY TESTING

Test Specimens

The test specimens represented a formation of homogeneous pink granite. All six specimens displayed a hypidiomorphic-granular, medium to coarse grained phaneritic texture. A hand lens examination revealed the composition to be approximately 45% potash feldspar, 20 to 25% plagioclase feldspar, 25 to 30% quartz, and 5 to 10% biotite plus accessory minerals.

The potash feldspar appeared as pink, anhedral to subhedral, nontransparent crystals. They were blocky in nature with their maximum dimension being commonly about 2 to 3 mm. A few potash feldspar crystals were observed to be about 10 to 30 mm long. Plagioclase feldspar occurred typically as clouded, white to colourless, anhedral to subhedral crystals, blocky to tabular in nature. Common grain sizes ranged from about 1.5 mm to about 3 mm with some larger grains in some specimens up to 8 mm long. Plagioclase grains often displayed polysynthetic twinning. Quartz was observed as clear, colourless, anhedral, equidimensional grains interstitial to the potash and plagioclase feldspar. Maximum grain sizes varied up to about 5 mm. Quartz crystals were usually clustered into larger aggregates. Anhedral flakes of dark brown to black biotite were seen interstitial to the other major rock forming minerals with minor muscovite and chlorite being associated with it in some instances.

None of the specimens showed any signs of structural weakness planes which might have an influence on strength values.

Test specimens were prepared as closely as possible to conform with ASTM standards (1,2). They were cut and machined into right circular cylinders, having length-to-diameter ratios of approximately 2.5. The sides of the specimens were smooth and free from abrupt irregularities. The specimen ends were ground flat and parallel to within 0.025 mm. Specimen diameters are given in Table 1.

The core specimens were tested in a surface dry condition and no efforts were made to maintain the field moisture condition of the rocks.

Procedures

After machining, each core specimen was fitted with three circumferential and three axial strain gauges of the electrical resistance type. They were equally spaced around the circumference of the specimens at mid-height. Each had a gauge length of 12.7 mm. During testing the averages of the three circumferential and the three axial strain measurements were continuously recorded with the axial load on a XYY recorder.

Prior to testing, the specimens were placed between hardened steel platens whose diameters were very close to those of the specimens. Impermeable flexible rubber membranes covered each specimen and neoprene O-rings were used to seal the specimen from the confining fluid in the triaxial cell.

The specimens were tested in a small standard triaxial compression cell capable of testing specimens up to approximately 64.1 mm in diameter under confining pressures up to approximately 55.2 MPa. The cell was placed in a uniaxial loading device of sufficient capacity to apply a continuous axial stress, without shock, at an approximately constant rate averaging 28.22 MPa/min. A thin teflon disc, 1.5 mm thick, was placed between one of the steel platens and the bearing block of the triaxial cell to reduce effects of non-parallelism. No spherical seats were used.

The confining pressure was applied to the specimen by means of a hydraulic pump capable of maintaining a constant lateral pressure during testing. Two tests were carried out at each of the three confining pressures of 3.45 MPa, 17.24 MPa and 34.48 MPa.

TABLE 1

Results of Triaxial Compression Tests on WN1 Granite Specimens

Specimen No.	Diameter (mm)	Confining Pressure σ_3 (MPa)	Axial Stress Rate (MPa/min)	Total Axial Strain (%)	Triaxial Strength ($\sigma_1 - \sigma_3$) (MPa)	Tangent Modulus (10^3 MPa)	Poisson's Ratio	Failure Characteristics
WN1 323.70	44.93	34.48	28.09	0.70	552.61	68.74	0.34	- Violent, 2 fragments - single shear plane, 69°
WN1 323.815	44.91	17.24	29.76	0.64	458.92	74.88	0.59	- violent, 3 fragments - bottom wedge, axial splitting
WN1 323.930	44.93	3.45	26.36	0.45	283.32	71.02	0.43	- violent, more than 3 fragments - bottom wedge, axial splitting
WN1 324.06	44.96	34.48	27.64	0.64	557.54	64.26	0.33	- violent, more than 3 fragments - bottom wedge, axial splitting
WN1 324.16	44.93	17.24	31.20	0.69	444.61	66.40	0.37	- violent, more than 3 fragments - double wedge
WN1 324.75	44.96	3.45	26.28	0.40	240.95	68.33	0.38	- violent, 2 fragments - single shear plane, 70°
mean std. dev.	44.94		28.22 1.94			68.94 3.70	0.41 0.10	

With a specimen in place and the cell assembled and filled with hydraulic fluid, a small axial load was applied to properly seat the bearing parts of the apparatus. Using a linear variable differential transformer to determine the position of the loading head, the confining pressure was then slowly raised to one of the predetermined test levels, while, at the same time, the axial load was increased to prevent the loading head from lifting. When the predetermined confining pressure was reached, the axial load bearing on the test specimen was considered to be the zero or starting load for the test. The axial load was then increased as previously described until specimen failure. Failure times ranged from approximately 9 min (lowest confining pressure) to approximately 20 min (highest confining pressure).

Results

The results of the triaxial compression tests are given in Table 1. The triaxial strengths, $\sigma_1 - \sigma_3$, were determined as follows:

$$\sigma_1 - \sigma_3 = \frac{P}{A}$$

where: P = net applied axial load (total load in axial direction minus the upward load resulting from the confining pressure)

A = cross-sectional area of specimen

Stress difference versus axial strain curves for the six tests are shown in Fig. 1. As can be seen the maximum axial strain undergone by a single specimen was 0.70% at the confining pressure of 34.48 MPa. For the three confining pressures the average triaxial strengths are listed below:

σ_3 (MPa)	$\sigma_1 - \sigma_3$ (MPa)
3.45	262.14
17.24	451.77
34.48	555.08

The increase in triaxial strength with confining pressure is shown in Fig. 2. Fivefold and tenfold increases in confining pressure resulted in strength increases of 1.7 fold and 2.1 fold respectively.

Both the tangent modulus of elasticity and Poisson's ratio were determined for each specimen at the 50% ultimate stress level from the axial and circumferential strain data that was recorded. The tangent modulus is represented

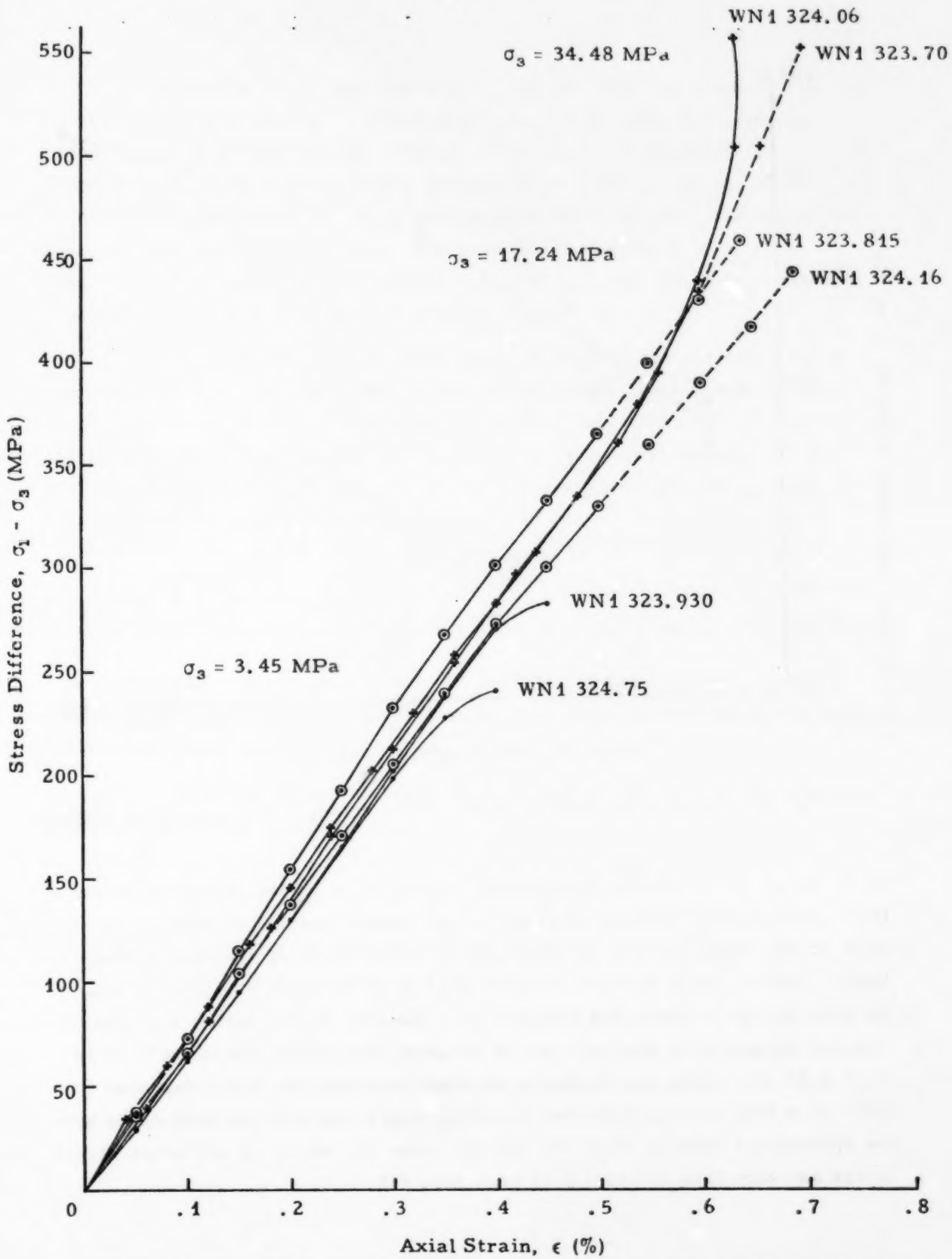

Fig. 1 Stress-strain curves for WN1 granite specimens at various confining pressures.

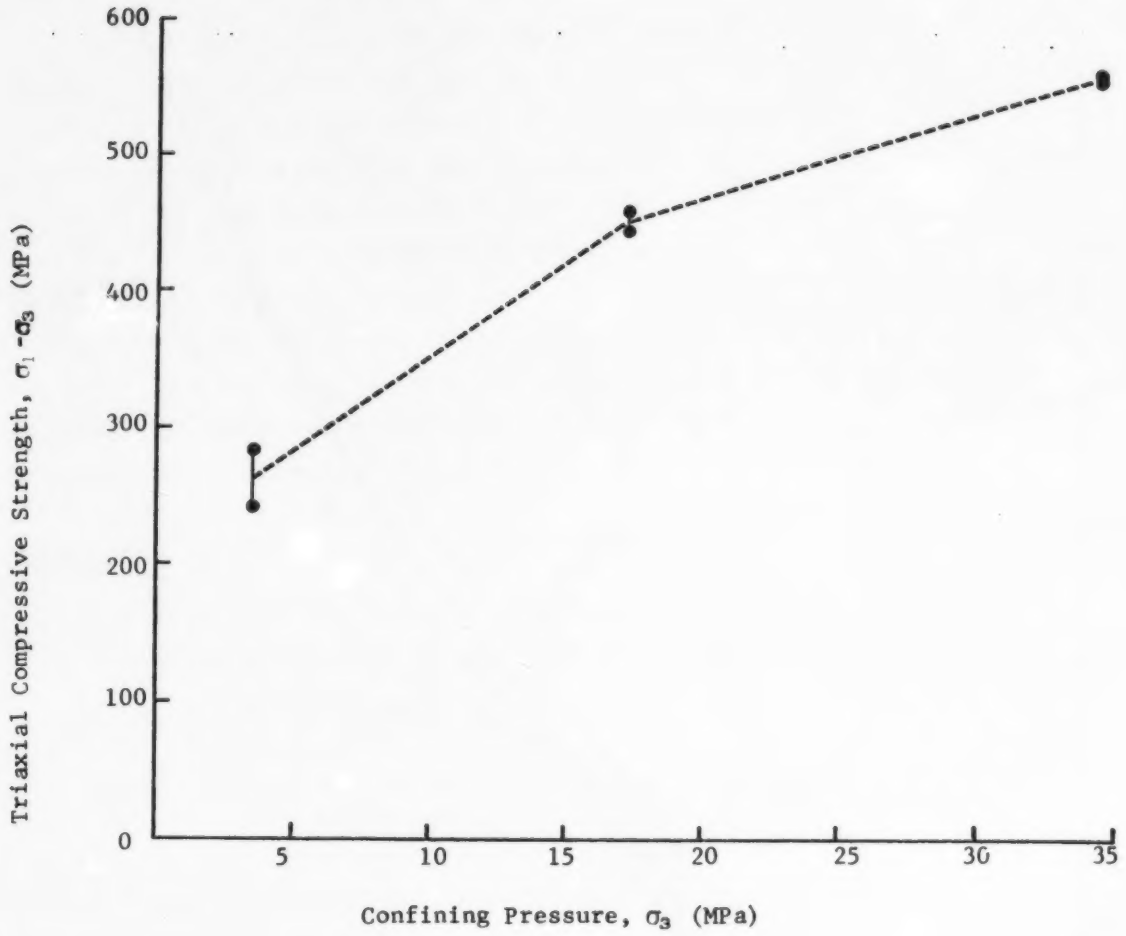

Fig. 2: Increase in triaxial compressive strength with confining pressure.

by the slope of the stress versus axial strain curve at the indicated stress level while Poisson's ratio is given by the tangent modulus divided by the slope of the stress versus circumferential strain curve at the same stress level. Table 1 shows the mean tangent modulus to be $68.94 \pm 3.70 \times 10^3$ MPa. The mean Poisson's ratio was 0.41 ± 0.10. These Poisson's ratios are high compared to generally accepted values for granite ranging from about 0.15 to about 0.24 (3). Lama and Vutukuri, however, indicate that the Poisson's ratio of a rock is very dependent upon the stress level and is influenced by the opening and closing of cracks and that measured values at different stress levels may vary from 0.1 to 1.0 or even more (4).

A Mohr's circle was constructed for each test as shown in Fig. 3. A "best-fit" Mohr envelope (curved dashed line) was drawn approximately tangent to the stress circles. Values of the angle of internal friction, $\emptyset$, were determined by constructing a tangent to the Mohr circle for each confining stress at the point of contact with the envelope. The angle this tangent made with the horizontal was reported as the angle of internal friction. The corresponding cohesion intercept, C, was determined for each tangent. The $\emptyset$ and C values are given in Fig. 3.

Two distinct failure modes were observed in the six tests which were carried out. Two specimens failed along single shear planes making angles of about 70° to the plane of the major principal stress. These failure plane angles are confirmed in the stress circles as there exists a fixed relationship between the location of the failure plane and the angle of friction of the rock. This relationship is:

$$\theta = 45° + \frac{\emptyset}{2}$$

where θ = angle between failure plane and plane of major principal stress.

The other mode, typical to fracture of brittle homogeneous rock, involved wedge failure with associated axial splitting. All failures were of a violent nature and their characteristics are reported in Table 1.

Shear and normal stresses on the plane of failure can be calculated from the following equations (1):

$$\tau\theta = (\sigma_1 - \sigma_3) \text{ Sin } \theta \text{ Cos } \theta, \text{ and}$$
$$\sigma\theta = \sigma_3 + (\sigma_1 - \sigma_3) \text{ Cos}^2 \theta,$$

where: $\tau\theta$ = shear stress on failure plane
$\sigma\theta$ = normal stress on failure plane

Graphically, the point of tangency of the strength envelope with a given stress circle (Fig. 3) represents the state of stress ($\sigma\theta$, $\tau\theta$) along a plane within the test specimen at time of failure. These stress values for the three confining pressures are given in Fig. 3.

Confining Pressure σ_3 (MPa)	Internal Friction Angle ϕ	Cohesion C (MPa)	Failure Plane Angle θ	Failure Plane Stresses (MPa)	
				$\sigma\theta$	$\tau\theta$
3.45	$\phi_1 = 64°$	$C_1 = 20$	$\theta_1 = 77°$	16.7	57.5
17.24	$\phi_2 = 54°$	$C_2 = 50$	$\theta_2 = 72°$	60.4	132.8
34.48	$\phi_3 = 45°$	$C_3 = 80$	$\theta_3 = 68°$	112.4	192.8

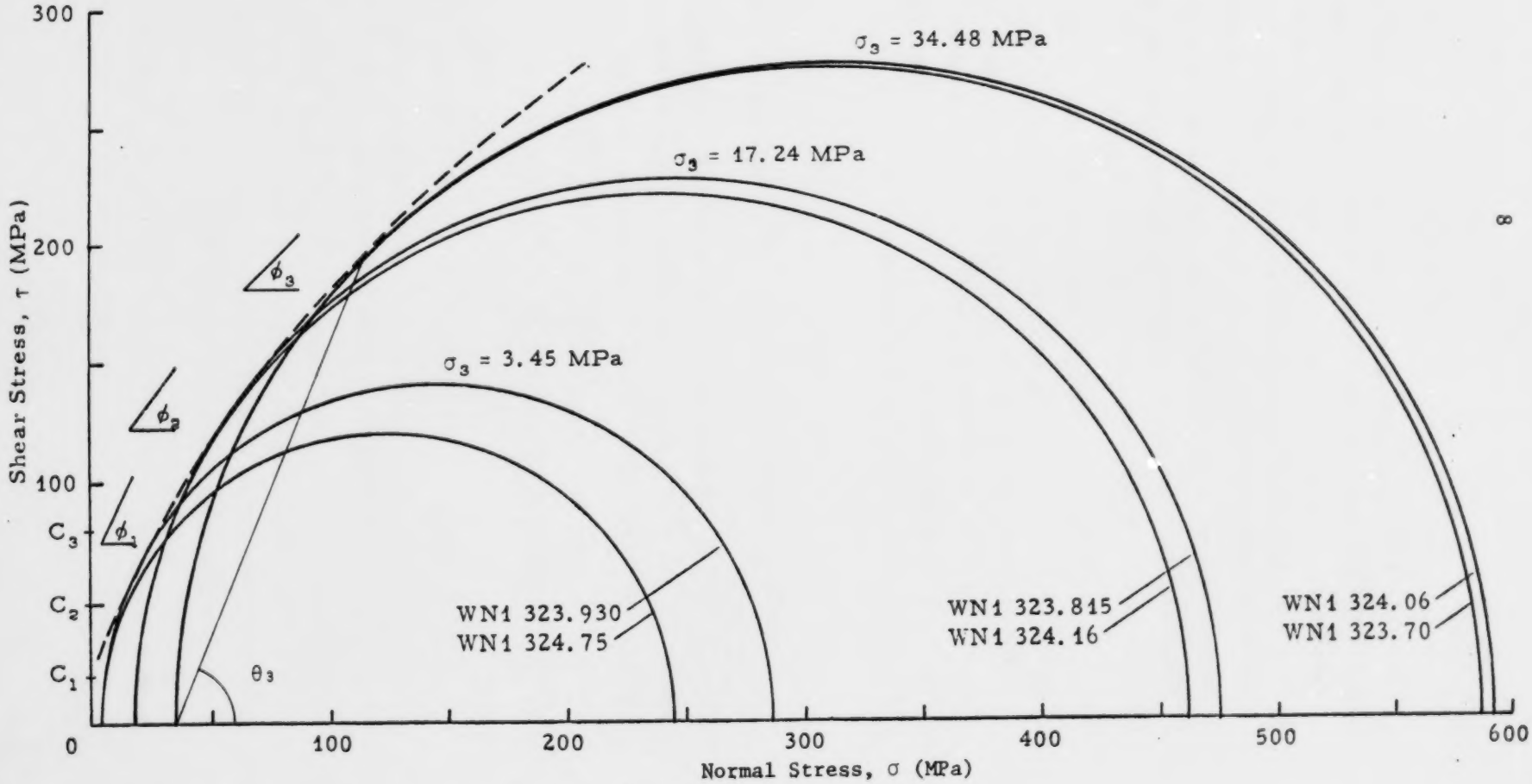

Fig. 3: Mohr's circles and the strength envelope.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the assistance of L. Tirrul in carrying out the testing program.

REFERENCES

1. "Standard test method for triaxial compressive strength of undrained rock core specimens without pore pressure measurements"; Designation: D2664-67 (reapproved 1974); American Society for Testing and Materials; Philadelphia, Pa; pp 327-330; 1974.

2. "Standard method of test for elastic moduli of rock core specimens in uniaxial compression"; Designation: D3148-72; American Society for Testing and Materials; Philadelphia, Pa; pp 404-407; 1973.

3. Krynine, D.P. and Judd, W.R. "Principles of engineering geology and geotechnics"; New York; McGraw-Hill; 1957.

4. Lama, R.D. and Vutukuri, V.S. "Handbook on mechanical properties of rocks"; vol II; Clausthal, Germany; Trans Tech Publications; 1978.